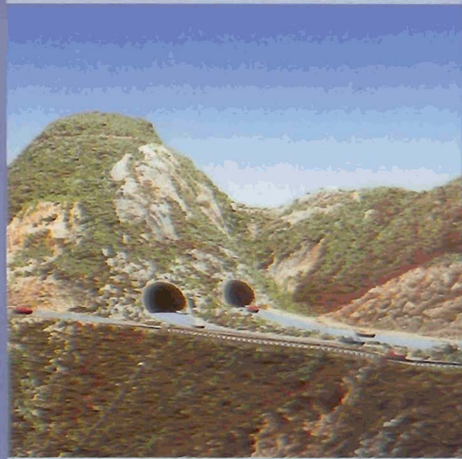
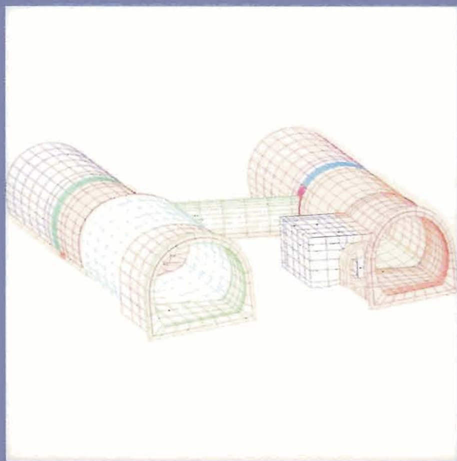
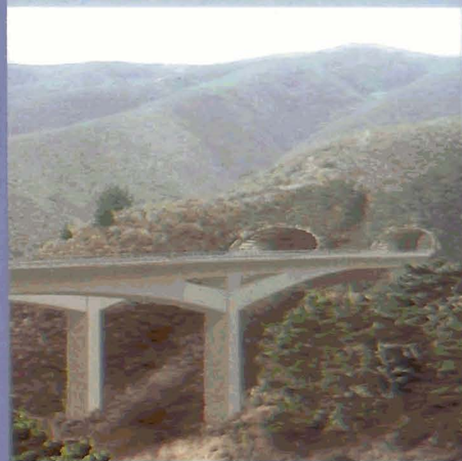
Caltrans

Devil's Slide Tunnels
STRUCTURAL DESIGN CRITERIA

San Mateo County District 4
EA 1123M1
Tunnel Contract
Kilometer Post: 61.47 to 62.04
March 15, 2005







HNTB

PART I. INITIAL SUPPORT

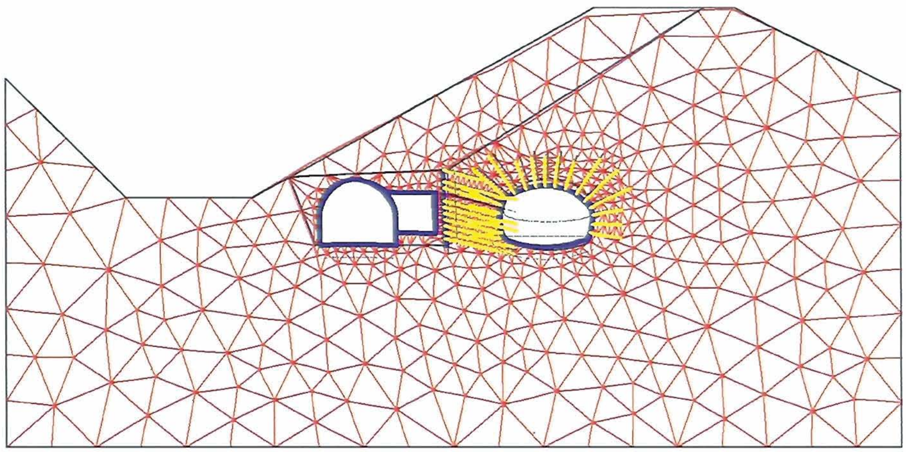
Ninth Draft

STRUCTURAL DESIGN CRITERIA

FOR

DEVILS SLIDE TUNNEL

PART I - INITIAL SUPPORT



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January 28,2005

RECORD of REVISIONS

Date of Issue	Description
3/17/04	First Draft
5/14/04	Second Draft
5/28/04	Third Draft
6/07/04	Fourth Draft
8/12/04	Fifth Draft
9/27/04	Sixth Draft
11/04/04	Seventh Draft
1/05/05	Eighth Draft
1/28/05	Ninth Draft

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SECTION 1

INTRODUCTION

1.1 Objective and Scope

The purpose of the ***Structural Design Criteria for Devils Slide Tunnel*** **–Initial** Support ***(Initial Support Criteria)*** is to provide technical background information, guidelines, and requirements for the structural analysis and design of the initial support for excavation of Devils Slide Tunnel located on State Route 1 south of the City of Pacifica in the county of San Mateo, California.

1.2 Design Approach

Initial Support Criteria assume tunnel construction will utilize a double shell lining system consisting of both initial support and final lining separated by a waterproofing and drainage system. The initial support includes steel fiber reinforced shotcrete and depending on in-situ ground conditions, can also include lattice girders, rock dowels, and other ground stabilizing techniques. The final lining will be constructed of cast-in-place reinforced concrete after the initial support and waterproofing have been placed. ***Initial Support Criteria*** are applicable to design of the initial support only. It will be assumed that the initial support deteriorates over time and all loads are transferred to and supported by the final lining.

Initial Support Criteria are based on the principles of the New Austrian Tunneling Method (NATM) assuming that the initial support elements act as a ring-like support structure.

1.3 Limits of Applicability

Initial Support Criteria for the initial support applies to the analysis and design of mined tunnels between cut-and-cover portals, cross passages connecting tunnels, and mined equipment chambers.

SECTION 2

CODES AND STANDARDS

2.1 Codes

The initial support is considered a temporary structure. Existing design codes such as the California Department of Transportation (Caltrans) ***Bridge Design Specifications (BDS)***, are therefore not applicable. Design of the initial support will therefore be governed by these criteria as well as referenced standards when appropriate.

2.2 Standards

The provisions of the ***Initial Support Criteria*** shall govern the design. Provisions in the following documents shall also be considered as guidelines when sufficient criteria are not provided by the ***Initial Support Criteria.***

(1) ASCE Technical Committee on Tunnel Lining Design: "Guidelines for Tunnel Lining Design", edited by T. O'Rourke, 1984
(2) ITA Working Group on General Approaches to the Design of Tunnels: "Guidelines for the Design of Tunnels", Tunneling and Underground Space Technology, Vol. 3, No 3, 1988
(3) ICE Design and Practice Guides: "Sprayed Concrete Linings (NATM) for Tunnels in Soft Ground, edited by Institution of Civil Engineers, 1996
(4) ACI, Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02), American Concrete Institute, Farmington Hills, Michigan, 2002

SECTION 3

STRUCTURAL MATERIALS

3.1 Shotcrete

Normal weight shotcrete shall be used and have the properties shown below. All shotcrete shall be steel fiber reinforced. When performing analysis, the modulus of elasticity of shotcrete shall be adjusted as shown in the table below which considers the age and characteristics of the shotcrete and the behavior of supported rock.

John M., Mattle B. (2003), *Shotcrete Lining Design: Factors of Influence, RETC* 2003 Proceedings, 726-734.

Shotcrete Lining: $f'_c = 28.0$ MPa (4000 psi)
Poisson's ratio: v = 0.2

Modulus of Elasticity		
Shotcrete Age/Strength	Rock Mass Behavior	
	Slow Stress Redistribution (Ductile Behavior)	Fast Stress Redistribution (Brittle Behavior)
Applied immediately after excavation [1-day-strength < 10 MPa (1500 psi)]	4000 – 6000 MPa (580 – 870 ksi)	4500 – 7000 MPa (650-1015 ksi)
Shotcrete obtaining design strength [$f'_c = 28.0$ MPa (4000 psi)] prior to the next excavation	15,000 MPa (2175 ksi)	15,000 MPa (2175 ksi)

3.2 Reinforcement

All reinforcement shall be ASTM A706 (A706M), grade 60, with the following specified properties:

Modulus of elasticity: $E_s = 200{,}000$ MPa (29,000 ksi)
Specified minimum yield stress: $f_y = 410$ MPa (60 ksi)

3.3 Rock Dowels

Rock dowels may be used to stabilize key blocks or to strengthen the rock mass and to provide supplement initial support of the initial lining. An elastic-perfectly plastic material model may be used in finite-element models where rock dowels are assumed to yield after reaching a prescribed yield strength.

3.4 Steel Arches

Steel arches (lattice girders), used as immediate support to protect miners and to define the profile, may be considered as reinforcement in the shotcrete lining if their spacing does not exceed 460 mm (18 inches).

3.5 Forepoling

3.5.1 Steel Spiles

Steel spiles may be used for forepoling to prevent breakouts at the tunnel face after excavation prior to the application of other support measures. The following types of spiles may be used:

- Drill and grout spiles
- Self-drill and grout spiles

The following material properties shall apply:

Minimum Yield Stress:	$f_y = 241$ MPa	(35 ksi)
Modulus of elasticity:	Es = 200,000 MPa	(29,000 ksi)

3.5.2 Steel Pipes

When steel spiles are not adequate, steel pipes may be used as an umbrella or canopy for forepoling.

The following material properties shall apply:

Minimum Yield Stress:	$f_y = 241$ MPa	(35 ksi)
Modulas of elasticity:	Es = 200,000 MPa	(29,000 ksi)

SECTION 4

DESIGN LOADS AND LOAD FACTORS

4.1 Design Loads

4.1.1 Structural Dead Loads

Structural dead loads of the initial support can be ignored since shotcrete, rock dowels, steel arches, etc. form a composite structure with the surrounding rock mass.

4.1.2 Live Loads

Live loads on the initial support need not be considered.

4.1.3 Rock Loading on Initial Support

All rock loading on the initial support shall be determined through analysis as specified in Section 5.1 of these criteria.

4.1.4 Determination of Rock Loading Applied to Final Lining

Rock loading to be applied to the final lining will be determined by analysis methods described in ***Initial*** Support ***Criteria*** Section ***5.1.*** When rock loading consists of loading from discrete rock blocks, the rock load shall be determined by considering the geometry of the rock block which is formed by discontinuities in the rock mass, assuming long-term deterioration of rock dowels has occurred. Rock loads developed through stress distribution within the rock mass after deterioration of the initial support shall be determined through finite element analysis.

4.1.5 Hydrostatic Pressure

No hydrostatic pressure shall be applied to the initial support. Water pressure will be released through the working faces of the excavation and through perforations in the initial lining.

4.1.6 Seismic Loads

The initial support is considered a temporary structure and no seismic loading is therefore required. The seismic performance of the initial support should be adequate for minor seismic events.

4.1.7 Thermal Forces

No thermal loading on the initial support is required.

4.2 Load Factors

4.2.1 Load Factors Applied to Initial Support

The following load factors, capacity reduction factors, or safety factors shall be applied to analysis results when designing the initial support:

Rock Dowels to Support Key Block Loading:

The product of the capacity reduction factors shown below shall be applied to the rock dowel capacity. No load factors shall be applied to the key block loading obtained through analysis.

$\varphi_{LT} = 0.79$ Type of loading reduction factor

$\varphi_{M} = 0.87$ Material reduction factor

φ_{FI} = See Table Field installation reduction factor

See Commentary for Section 4.3.1

Number of Rock Dowels per Key Block	φ_{FI}
1	0.50
2	0.67
3	0.75
4	0.80
5	0.85
>6	0.90

Rock Dowels used to Supplement Initial Support:

Tensile loads on rock dowels obtained through finite element analysis shall not be factored. A factor of safety of 1.6 shall be applied to the yield strength of the rock dowels when determining the size and number of rock dowels required to resist loading indicated by analysis.

Rock Dowels used to Provide Face Stability:

A factor of safety of 1.6 shall be applied to the yield strength of rock dowels when rock dowels are required to assist with the face stability of the excavation heading as determined in three-dimensional face stability calculations.

Shotcrete Initial Lining:

The nominal moment capacity of the initial lining shall be determined in accordance with ACI 318-02 considering both the axial load and bending moment on the lining. All loading determined through analysis shall be multiplied by a load factor of 1.4 when determining design section forces. A capacity reduction factor of 0.70 shall be applied when determining the capacity of the lining section.

4.2.2 Load Factors Applied to Final Lining

In recognition of the conservative assumption of the full deterioration of the initial support requiring the final lining to support all loading, rock loading on the final lining shall be considered upper bound and load factors shall not be applied to these rock loadings.

SECTION 5

ANALYSIS ASSUMPTIONS

5.1 Analyses Methods

5.1.1 General

Analyses methods for the initial support shall consider the following modes of failure of the rock mass and corresponding analysis procedures:

- Failure of discrete rock blocks shall be analyzed through "wedge" analysis.
- Failure of the rock mass through either stress or discontinuity induced fracturing, progressive failures induced by high stresses, or stress induced failures originating ahead of the tunnel face, shall be analyzed by the finite element method.
- Failure of the tunnel face through slope instability shall be analyzed by three-dimensional wedge stability analysis.

5.1.2 Wedge Analysis

The computer program UNWEDGE shall be used to determine key block loading and to select the number and length of rock dowels required to stabilization the key block. Key block sizes will be defined according to the geometry of discontinuities and persistency of joints.

UNWEDGE V2.37 or higher by RocScience, Inc. 31 Balsam Avenue, Toronto, Canada M43 3B5

5.1.3 Finite Element Method

The finite element method shall be used to assess the state of stress and deformation in the initial support reinforced excavation. The analysis shall consider the construction sequence used to excavate the tunnel and install the initial support. Beam-continuum models shall be used with the rock mass represented with continuum finite elements and the initial lining represented with beam elements. Interface elements shall be used between rock continuum and lining elements. Soil continuum elements may be represented by a elastic-plastic Mohr-Coulomb material models, or other appropriate material models as required by site specific conditions. Either linear-elastic or elastic-perfectly plastic material models may be used for the beam elements representing the initial lining. Appropriate reductions to the modulus of elasticity of the initial lining beam elements shall be applied as specified in ***Initial*** Support ***Criteria*** Section 3.1. If rock dowels are used as part of the initial support, they shall be represented in the model by tension-only truss elements. Either a linear-elastic or elastic-perfectly plastic material model may be used for rock dowel truss elements.

Either two or three-dimensional finite element models may be used. Since three-dimensional arching of the rock mass will occur as the excavation advances, appropriate adjustments should be made to two-dimensional models to capture three-dimensional effects.

Kielbassa and Duddeck (1991), "Stress-Strain Fields at the Tunneling Face – Three-dimensional Analysis for Two-dimensional Technical Approach", Rock Mechanics and Rock Engineering, Springer Verlag.

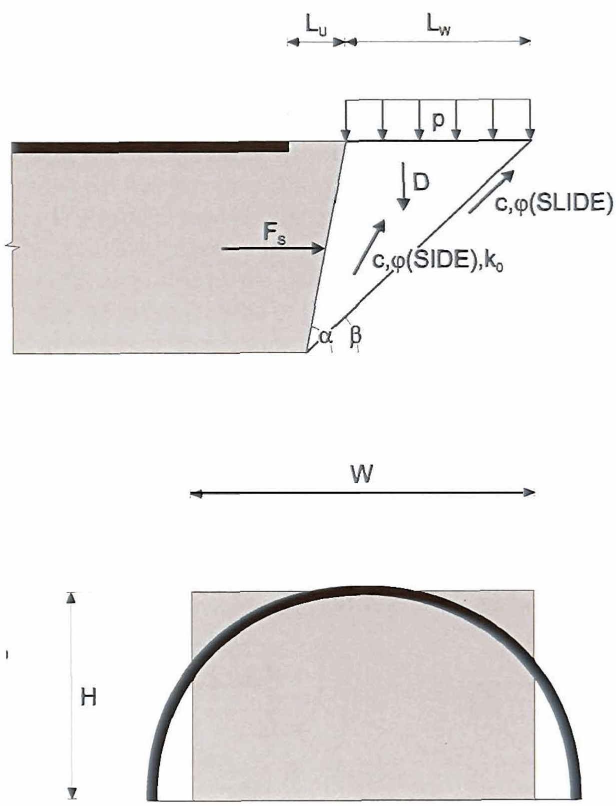
Schikora K., Fink T. (1982). Caluculation methods commonly used in subway mining. Civil Engineer (German), 57, 193-198

5.2 Face Stability for Jointed Rock

During excavation of the tunnel, wedges may slide from the face of the excavation into the tunnel opening where joints act as sliding planes. This failure mode shall be investigated by using a three-dimensional calculation model. The excavated section can be assumed as a rectangle with the height of excavation (H) and an equivalent width (W). The forces that act on the wedge are separated into driving forces that produce failure and resisting forces that provide stability.

The following assumptions are used to determine face stability. See "Figure Showing Face Stability Geometry" for a definition of parameters.

- Driving forces result from the self-weight of the sliding wedge and any additional load acting on the wedge. The additional load is derived from maximum block sizes according to wedge analysis when applicable or from the silo theory if the excavation is in loose ground.
- Resisting forces are provided by friction and cohesion on both the sliding plane and the side planes. The frictional forces on the side planes of the wedge are calculated using the weight of the wedge multiplied by the horizontal pressure coefficient.
- The resisting force shall be at least 1.4 times greater than the driving force. If required, rock dowels shall be added at the face of the excavation to increase the resisting force.



Where:

L_u = Unsupported length
L_w = Length of wedge
H = Height of excavation
W = Equivalent excavation width
P = Loading from loose ground
D = Dead load of wedge

α = Slope of tunnel face
β = Slope of slide plane
c = Rock cohesion
ϕ = Rock angle of internal friction
k_o = Horizontal to vertical stress ratio

Figure Showing Face Stability Geometry

5.3 Forepoling

When forepoling is required to stabilize the heading of the tunnel excavation, it shall be assumed that the forepoling does not form a closed ring around the tunnel to carry ground load similar to the initial lining, but acts as a longitudinal beam when providing excavation support. One end of the forepoling can be assumed to be supported by the shotcrete lining, while the other end is supported by the ground ahead of the excavation face. Forepoling shall be designed as beams, with a assumed span of 1.5 times the unsupported length with fixity assumptions provided in the figure below. The load p_1 acting on forepoling shall be arrived at by considering site-specific rock properties applied to appropriate references.

Mahtab, M. A. and Grasso, P. "Geomechanical Principles in the Design of Tunnels and Caverns in Rock", Elsevier, 1992, Figure 2.11.

John, M. and Mattle, B. (2003), Design of Tube Umbrellas, Tunnel 11, RŎCNIK, č. 3/2002, 2-9.

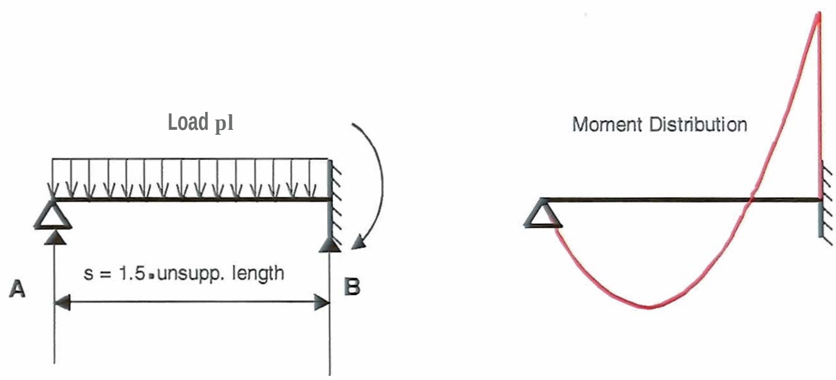


Figure Showing Structural Model of Forepoling

SECTION 6

COMMENTARY

4.3.1 Rock Loading on Initial Support

Rock dowels used to provide support for key blocks can be loaded by tensile and shear forces. For in situ conditions, it is difficult to determine the actual tensile and shear forces on the rock dowels. Therefore, it is assumed that rock dowels are loaded 50% in tension and 50% in shear. A capacity reduction factor of 0.58 is applied to the rock dowel shear capacity according to the Mise's hypothesis given by:

$$\tau = \frac{\sigma_t}{3^{1/2}} = 0.58\ \sigma_1$$

Where: σ_1 = tensile strength

τ = shear strength

No reduction factor is applied for tensile capacity of the rock dowels. This results in a combined reduction factor of φ_{LT} = 0.79. Note: (0.58 x 0.5) + 0.5 = 0.79

PART II. FINAL LINING AND PORTALS

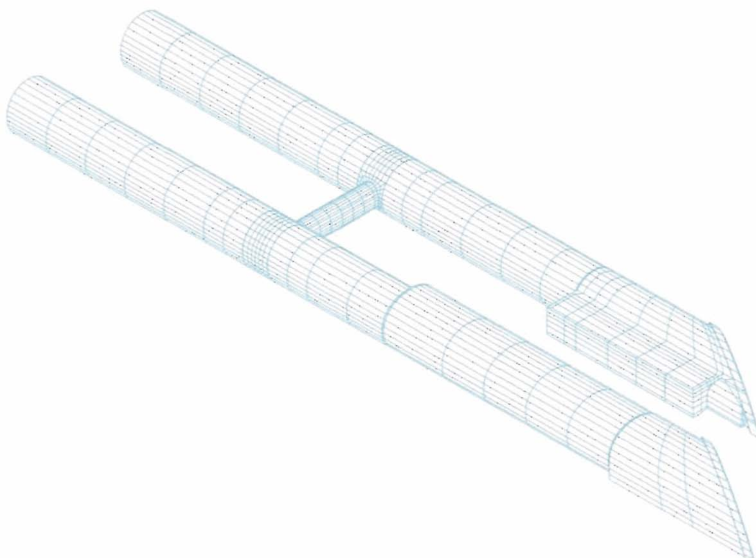
Fourth Draft

STRUCTURAL DESIGN CRITERIA

FOR

DEVILS SLIDE TUNNEL

PART II - FINAL LINING AND PORTALS



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December 31,2004

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SECTION 1

INTRODUCTION

1.1 Objective and Scope

The purpose of the *Structural Design Criteria for Devils Slide Tunnel (Criteria)* is to provide technical background information, guidelines, and requirements for the structural analysis and design of the tunnel final lining and cut-and-cover portals of Devils Slide Tunnel located on State Route 1 south of the City of Pacifica in the county of San Mateo, California.

1.2 Design Approach

The Load Factor Design (LFD) Method will be used for design of all concrete and steel structural members. Rock loads will not be factored as they will be assumed to represent upper bound limits and will be verified by geotechnical observation during tunnel construction. Design considers ultimate limit state for strength as well as serviceability checks for deflections and concrete cracking widths.

Criteria assumes tunnel construction will utilize a double shell lining system consisting of both initial support and final lining separated by a waterproofing and drainage system. The initial support includes steel fiber reinforced concrete and depending on in-situ ground conditions, can also include lattice girders, rock dowels, and other ground stabilizing techniques. The final lining will be constructed of cast-in-place reinforced concrete after the initial support and waterproofing have been placed. *Criteria* is applicable to the design of the final lining only. It will be assumed that the initial support deteriorates over time and all loads are transferred to and supported by the final lining.

1.3 Limits of Applicability

Criteria applies to the analysis and design of the main tunnel portal to portal, cross passages between tunnels, underground equipment rooms, and cut-and-cover portals. *Criteria* does not apply to the following components of the project:

- Approach structures or appurtenant structures not attached to the tunnel such as the Operations and Maintenance Building.
- Equipment and utilities supports or the mounting of equipment and utilities supports to the tunnel final lining.
- Stabilization of rock slopes above tunnels.

SECTION 2

CODES AND STANDARDS

2.1 Codes

The design of the tunnel shall conform to the California Department of Transportation (Caltrans) ***Bridge Design Specifications (BDS)***, except as modified or augmented by the ***Criteria.***

2.2 Standards

The provisions of the ***Criteria*** and Caltrans BDS shall govern the design. Provisions in the following documents shall also be considered as guidelines when sufficient criteria are not provided by either BDS or ***Criteria.***

(1) 2002 Interim AASHTO, ***LRFD Bridge Design Specifications,*** 2nd edition – 1998, American Associations of State Highway and Transportation Officials, Washington, DC, 2002
(2) ACI, Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02), American Concrete Institute, Farmington Hills, Michigan, 2002
(3) Caltrans, ***Bridge Memo To Designers ,*** California Department of Transportation, Sacramento, California, 1998
(4) Caltrans, ***Bridge Design Aids Manual,*** California Department of Transportation, Sacramento, California, 1995
(5) Caltrans, ***Seismic Design Criteria,*** Version 1.2, California Department of Transportation, Sacramento, California, 2001
(6) AWS, Bridge Welding Code, ANSI/AASHTO/AWS D1.5-95, American Welding Society, Miami Florida, 1995

SECTION 3

STRUCTURAL MATERIALS

3.1 Structural Steel

Structural steels shall conform to AASHTO designation M 270M, Grade 250, 345, 485W, or 690 (M270, Grade 36, 50, 70W, or 100) (ASTM designation A709). The material properties shall be as specified in Article 6.4.1 of the AASHTO-LRFD Specifications.

Welds shall conform to specifications in the Bridge Welding Code ANSI/AASHTO/AWS D1.5-95.

High strength bolts shall conform to AASHTO M 164M (M164) (ASTM A325). Their minimum tensile strength shall be as specified in Article 6.4.3 of AASHTO-LFRD. All new bolts shall be galvanized. AASHTO M 253M (M253) (ASTM A490) bolts and Direct Tension Indicators are not permitted.

3.2 Structural Concrete

3.2.1 Concrete

Normal weight concrete shall be used and have the properties shown below. The capacity of concrete components to resist all seismic effects, except for shear, shall be based on most probable (expected) material properties to provide a more realistic estimate for design strength and seismic response.

Seismic Design Criteria, Section 3.2

Final Lining:	$f'_c = 28.0$ MPa	(4000 psi)
	$w_c = 23.6$ kN/m^3	(150 pcf)
	[May be increased to 34.5 MPa (5000 psi) if warranted by design]	
Portal:	$f'_c = 28.0$ MPa	(4000 psi)
	$w_c = 23.6$ kNlm^3	(150 pcf)
	[May be increased to 34.5 MPa (5000 psi) if warranted by design]	

See Commentary for Section 3.2.1.

Modulas of elasticity:	$E_c = w_c^{1.5} 0.043\sqrt{f'_{ce}}$ MPa	($w_c^{1.5} 33\sqrt{f'_{ce}}$, psi)
Shear Modulas	$G_c = Ec/(2x(1+v_c))$	
Poisson's ratio:	$v = 0.2$	
Modulas of rupture:	$f_r = 0.63\sqrt{f'_c}$, MPa	($7.54 f'_c$, psi)
Where:	f'_c = 28 day compression strength	
	w_c = density of concrete	

Seismic Design Parameters:

Expected concrete compressive strength:	$f'_{ce} = 1.3 \times f'_c = 36.4$ MPa	(5200 psi)

Unconfined Concrete compression strain at maximum compressive stress: $\varepsilon_{co} = 0.002$

Ultimate unconfined compression (spalling) strain: $\varepsilon_{sp} = 0.005$

When sufficient ties are provided to effectively confine the concrete section, the confined concrete ultimate strength, strain at ultimate concrete strength, and ultimate concrete strength shall be determined by a constitutive stress strain model for confined concrete.

Mander et. al. Journal of Structural Engineering, American Society of Civil Engineers, 1988, pg 1804-1849.

3.2.2 Reinforcement

All reinforcement shall use ASTM A706 (A706M), grade 60, with the following specified properties:

Modulas of elasticity:	Es = 200,000 MPa	(29,000 ksi)
Specified minimum yield stress:	f_y = 410 MPa	(60 ksi)
Nominal yield strain:	$\varepsilon_y = 0.0021$	

Seismic Design Parameters:

Expected yield stress:	f_{ye} = 470 MPa	(68 ksi)
Specified minimum tensile stress:	f_u = 550 MPa	(80 ksi)
Expected tensile strength:	f_u = 655 MPa	(95 ksi)
Specified maximum yield stress:	f_{ymax} = 550 MPa	(80 ksi)
Expected yield strain:	$\varepsilon_y = 0.0023$	
Strain hardening strain ε_{sh}:	0.0150 #25 (#8) bars and smaller 0.0125 #29 (#9) bars 0.0115 #32 (#10) and #36 (#11) bars 0.0075 #43 (#14) bars 0.0050 #57 (#18) bars	
Ultimate tensile strain, ε_{su}:	0.120 #32 (#10) bars and smaller 0.090 #36 (#11) bars and larger	

SECTION 4

DESIGN LOADS AND GROUP LOADING

4.1 Structural Dead Loads

Structural dead loads of structural and non-structural elements shall be based on unit weights and computed volume of the materials. The following unit weights shall be used:

Unreinforced Concrete	2400 kg/m^3	(150 pcf)
Reinforced Concrete	2400 kg/m^3	(150 pcf)
Structural Steel	7850 kg/m^3	(490 pcf)
Timber	960 kg/m^3	(60 pcf)
Water	1000 kg/m^3	(62.4 pcf)
Saturated Earth Backfill	2240 kg/m^3	(140 pcf)
Bituminous Substances	2080 kg/m^3	(130 pcf)
Equipment		
Jet Fan:	1361 kg	(3000 lbs)
Variable Message Signs:	395 kg	(870 lbs)
Camera Assembly:	26 kg	(58 lbs)

4.2 Live Loads

Design live load shall consist of any non-permanent load placed on or in the tunnel. Where vehicles can gain access above the tunnel and the depth of fill over the crown of the tunnel is 3-m (10-ft) or less, the tunnel shall be design for HS20-44 loading. HS20-44 loading shall also be applied to the tunnel invert. Live load distribution shall be in accordance with BDS. Other live loading to consider include the following:

BDS, Section 6.4

Live Load Above Tunnel When Cover Is Less Than or Equal To 3-m (10-ft) (Not to be combined with HS20-44 loading):	9.6 KPa (200 psf)
Walkways:	4.8 KPa (100 psf)
Mechanical Systems (crown area only)	0.2 KPa (5 psf)

4.3 Wind Loads

Winds loads need not be applied to the tunnel.

4.4 Rock and Earth Pressure Loads

It is assumed that the tunnel's initial lining deteriorates under long term conditions and all geostatic pressure is carried by the final lining only. Rock loads for the tunnel's final lining and earth pressures for the cut-and-cover tunnels are provided in the following reports:

Report	Date	Title
ad_30_rev1_final	7-02-2004	Initial Support Calculations and Determination of Rock Loads Part 1 – South Block Entrance Section
ad_31_rev2_final	7-14-2004	Initial Support Calculations and Determination of Rock Loads Part 2 – Fault A Sections
ad_32_rev2_final	7-07-2004	Initial Support Calculations and Determination of Rock Loads Part 3 – Fault B Sections
ad_33_rev2[1]	8-13-2004	Initial Support Calculations and Determination of Rock Loads Part 4 – South Block Sections
ad_35_rev1-1[1] & ad_35_rev1-2[1]	8-12-2004	Initial Support Calculations and Determination of Rock Loads – Central Block Sections
ad_36_rev0-1 & ad_36_rev0-2	8-23-2004	Initial Support Calculations and Determination of Rock Loads – North Block – Part 1
ad_39_rev0	8-27-2004	Initial Support Calculations and Determination of Rock Loads – North Block – Part 2
ad_40_rev0	9-14-2004	Initial Support Calculations and Determination of Rock Loads – Cross Passages for Pedestrians
ad_41_rev0	9-17-2004	Determination of Rock Loads for Junctions of South and Central Equipment Chamber
ad_027 & 034	9-23-2004	Geotechnical Design Report

4.5 Hydrostatic Pressure

A waterproofing and drainage system will be provided between the initial support and final lining. Therefore, no hydrostatic pressure will be applied to the final lining. The waterproofing and drainage system shall be design to prevent the development of hydrostatic pressures throughout the life of the structure.

4.6 Seismic Loads

See Section 8 for seismic loading.

4.7 Thermal Forces and Concrete Shrinkage

4.7.1 General

Thermal forces and concrete shrinkage shall be considered in the final lining only. As a consequence of thermal variations and drying shrinkage, the final lining will experience strains and stresses due to expansion and contraction. A constant change in the lining temperature will cause changes in member length along the circumferential length of the lining. A change in temperature through the thickness of the final lining from thermal gradients can cause flexural stresses in the lining. Concrete shrinkage strains will be assumed to be constant across the entire cross section.

BDS Section 3.16

4.7.2 Thermal Forces

The final lining shall be designed for thermal forces. Cut-and-cover portals need not be designed for thermal forces.

Temperature changes shall be in accordance to Caltrans' **BDS** which mandates a rise or fall of 17" C (30° F) for concrete structures in a mild costal area. These temperature extremes shall be assumed applicable to portions of the tunnel final lining adjacent to portals. Recognizing that temperature variations within the tunnel away from portals will be more moderate, portions of the tunnel final lining at least 200 m (656.2 ft) from a portal face may be designed for 213 of these temperature extremes. The coefficient of thermal expansion shall be assumed to equal 11E-06 mm/mm/°C (6.0E-06 in/in/°F).

In addition to thermal expansion and contraction, the final lining shall be investigated for effects of the following thermal gradient across the thickness of the lining:

Location	Construction	Summer	Winter
Outer Surface	16" C (60° F)	16" C (60° F)	10" C (50° F)
Middle Surface	16° C (60° F)	21.5° C (70" F)	7° C (45° F)
Inner Surface	16° C (60° F)	27" C (80° F)	4.5" C (40" F)

4.7.3 Shrinkage

Caltrans' **BDS** value of 0.0002 for shrinkage shall be used. The shrinkage value is expressed in terms of temperature change as:

$T = \varepsilon_{sh}/\alpha_T = -18.2\ °C$ (-33.3 °F)

Where a, (coefficient of thermal expansion) = 11E-06 mm/mm/°C (6.0E-06 in/in/°F)

4.8 Load Combinations

Load combinations applied to the tunnel lining and portals shall comply with the following:

$$\text{Group (N)} = \gamma[\beta_D D + \beta_{LL+I} LL{+}I + \beta_E E + \beta_{Te} T_e + \beta_{S+TC} S{+}T_C + \beta_{EQ} EQ]$$

Group[1]	Gamma Factor	Beta Factors					
		D	LL+I	E	T_e	$S+T_c$	EQ
I	1.3	1.0	1.67	β_e	0	0	0
Va	1.3	1.0	1.0	β_e	0	1.0	0
Vb	1.3	1.0	1.0	β_e	1.0	0	0
VII	1.0	1.0	0	β_e	0	0	1.0

β_e = 0.77 maximum, 0.0 minimum for rock loads on final lining without EQ
β_e = 1.0 maximum, 0.0 minimum for rock loads on final lining with EQ
β_e = 1.0 for cut-and-cover portals
[1]Groups Va and Vb do not apply to cut-and-cover portals.

Where:
N = Group number
D = Dead load
LL+I = Live load and impact force
E = Earth pressure and rock load
$S+T_c$ = Shrinkage and thermal loads (contraction)
T_e = Thermal load (expansion)
EQ = Earthquake loading

SECTION 5

ANALYSIS ASSUMPTIONS

5.1 Analyses Methods

5.1.1 General

Analysis may be made with either beam-continuum or beam-spring models. Linear-elastic analysis is adequate for static loading of the lining and portals from self weight, geo-static, temperature, shrinkage, and live loads. Non-linear analysis shall be incorporated for seismic analysis if the lining or portals experiences significant non-linear response while responding to seismic induced deformations. Ovalization of the lining and racking of cut-and-cover portals shall be considered. The compressibility of the waterproofing system shall be accounted for.

5.1.2 Beam-Spring Models

Beam-spring models consists of representing the lining or portal by a series of beam elements. Either linear-elastic or non-linear beam elements can be used. When using linear-elastic beam elements, appropriate reductions in element stiffness due to concrete cracking shall be accounted for. The rock or soil medium is represented by radial and tangential springs. Where a waterproofing system is used between the initial support and final lining, tangential springs can be ignored. Loads and or displacements are predetermined and applied directly to the lining through the springs.

5.1.3 Beam-Continuum Models

Beam-continuum models consist of representing surrounding rock and soil with continuum finite elements. These continuum finite elements are assumed to be linear-elastic. Either linear-elastic or non-linear beam elements can be used to represent the lining. When using linear-elastic beam elements, appropriate reductions in element stiffness due to concrete cracking shall be accounted for. Interface elements are required between continuum and lining elements. Where a waterproofing system is used between the initial support and final lining, tangential interface elements can be ignored.

5.2 Final Lining

The final lining shall be analyzed for all self-weight and all other permanent static loading as well as transient loading from seismic activity. Analysis of the final lining shall be made under the assumption that the initial support does not contribute any resistance to superimposed loads and displacements. The initial support may be considered as integral with surrounding rock and soil and not to directly load the final lining, except as a portion of the permanent geostatic loading. Either beam-spring models or the beam-continuum models may be used to design the final lining.

5.3 Cut-and-Cover Portals

Cut-and-cover portals shall be analyzed for all self-weight and all other permanent static loading as well as transient loading from seismic activity. Either beam-spring models or the beam-continuum models may be used to design cut-and-cover portals.

5.4 Free Standing Portals

This section omitted after Draft 3.

5.5 Seismic Analysis

5.5.1 Pseudo-Static Time History Analysis

Seismic analysis of the final lining and cut-and-cover portals shall be defined in terms of induced displacements originating from the interaction of shear waves from the Maximum Credible Earthquake (MCE) with the tunnel. When subjected to these seismic shear waves, the tunnel lining and cut-and-cover portals will conform to these induced distortions by "ovaling" and "racking". During ovaling and racking, analyses shall make provisions for possible separation of the structure from the ground through the use of gap elements in series with the radial springs used at beam-spring models or similar modeling techniques at interface elements of beam-continuum models. However, due to the shape of the tunnel section, separation may be assumed to occur only across a limited portion of the lining. Under this assumption, it can also be assumed that dynamic response of the final lining and cut-and-cover portals will not occur when subjected to seismic displacements. Therefore, pseudo-static time history analysis may be used. Pseudo-static analysis shall consist of stepping the structure statically through displacement time history records.

5.5.2 Dynamic Time History Analysis

When any portion of the tunnel can respond dynamically, dynamic time history analysis shall be used. All analyses incorporating non-linear behavior when required by *Criteria* Section 8.3.3 shall be conducted using inelastic dynamic time history procedures. Non-linear inelastic dynamic time history analyses shall consider geometric nonlinearity (large displacements), non-linear boundary conditions, and inelastic member behavior.

5.5.3 Modeling Considerations

Two-dimensional models may be used to assess the behavior of the tunnel lining cross-section to racking and ovaling distortions imposed by the surrounding rock.

Where variations in either tunnel construction or geological conditions occur along the length of the tunnel, three dimensional continuum models shall be used to capture the response of the tunnel along three orthogonal axis. Examples of variation along the length that would require a three-dimensional analysis include the following:

- Transitions at portals
- Transitions to differing tunnel cross sections
- At intersections of cross passages
- Changes in ground motion due to both wave propagation and rock attenuation effects that result in adverse tunnel movement
- Change due to differing ground conditions

Appropriate boundary conditions shall be used to capture the interaction between the tunnel lining and surrounding rock. Non-linear springs shall be used when appropriate.

SECTION 6

CONCRETE DESIGN

6.1 Member Capacity

6.1.1 Flexural-Thrust Capacity

The nominal moment capacity M_n shall be calculated by considering the combined effects of axial and flexural loading. Compression axial load may be conservatively ignored when axial loads are 15% or less of the gross axial load capacity of the section under investigation. Gross axial load capacity is defined as the cross-section area of the section multiplied by the concrete compressive strength, f'_c. Flexural and axial capacities shall be calculated in accordance with BDS.

6.1.2 Shear Capacity

Seismic Design Criteria, Section 3.6

The nominal shear capacity V_n, to resist service loads, shall be in accordance with BDS. At the tunnel final lining, the shear capacity provided by concrete to resist service loads and seismic loads when the flexural Demand/Capacity (D/C) ratio is 1.0 or less, shall be in accordance with BDS section 8.16.6.2. At cut-and-cover portals, the shear capacity provided by concrete to resist service loads and seismic loads when the flexural Demand/Capacity (D/C) ratio is 1.0 or less, shall be in accordance with BDS section 8.16.6.7. If shear reinforcement is used, shear strength provided by the reinforcement shall be determined in accordance with BDS Section 8.16.6.3. If flexural seismic D/C ratios exceed 1.5, Seismic Design Criteria shall be used to determine V_n. If flexural seismic D/C ratios are larger than 1.0 but do not exceed 1.5, the following equation may be used to calculate the concrete shear capacity of both the final lining and cut-and cover portals:

$$\varphi V_c = \varphi 2.2\sqrt{f'_c}bd$$

Where: $\varphi = 0.85$

6.2 Member Shear Demand

For dead load and geo-static loading, design shear forces shall be the actual shear demands obtained from analysis. For Group VII seismic loading, shear demands shall be increased by an over-strength factor of 1.2 when the flexural D/C ratio is 1.0 or less. When the flexural D/C ratio is between 1.0 and 1.5, shear demands corresponding to flexural D/C ratio of 1.2 shall be used. When plastic hinges are introduced into models due to flexural D/C ratios exceeding 1.5, resulting shear demands shall be increased by an over-strength factor of 1.2

6.3 Design

6.3.1 Corrosion Protection

Corrosion protection will be accommodated by providing the following concrete cover over reinforcement:

Reinforcement at tunnel envelop	75 mm (3 in.)
Reinforcement at waterproofing	50 mm (2 in.)
Crossties, both faces final lining and portals	62 mm (2.5 in.)
Abutments and footings	75 mm (3 in.)
Curbs and railings	30[(a)] mm (1.18 in.)

[(a)]Use pre-fabricated epoxy coated reinforcing bars (ECR).

The South Portal and appurtenant tunnel are within 305 m (1000 ft) of the ocean. Discussions with Caltrans led to the decision to proceed with corrosion protection for a "Marine Environment" as specified in BDS Table 8.22.1 for the entire tunnel, portal to portal.

6.3.2 Distribution of Flexural Reinforcement

Control of flexural cracking shall conform to BDS Section 8.16.8.4. A z factor of 22.8 kN/mm (130 kips/inch) shall be complied with.

See Commentary for Section 6.3.2.

6.3.3 Minimum Reinforcement – Final Lining

Final Lining
Reinforcement for the final lining shall not be less than 0.003 times the gross concrete area in both the longitudinal and transverse direction of the final lining. Reinforcement shall be continuous or properly lapped spliced, and distributed uniformly across the lining section. Spacing of reinforcing bars shall not exceed 150 mm (6 in.). Reinforcing bar size shall preferably be limited to a #19 (#6) bar or smaller.

Cut-and-cover Portals
At cut-and-cover portals, main flexural reinforcement shall not be less than the lesser of 0.004 or 1.33 times the amount required by ultimate strength design. The minimum area of longitudinal reinforcement shall be 0.002 times the gross concrete area for slabs and 0.0025 times the gross concrete area for walls. Minimum longitudinal reinforcement area need not exceed 16,732 mm^2/m (0.79 in^2/ft) placed at each face regardless of the thickness of the wall or slab.

6.3.4 Minimum Thickness of Tunnel Final Lining and Portal Slabs and Walls

To allow for proper concrete placement and consolidation, accommodate the crown slick line for pumping concrete, the minimum liner thickness shall not be less than the following:

Two layers of reinforcement:	375 mm	(15.7 in.)

Walls and slabs of cut-and-cover portals and the tunnel final lining shall contain two layers of reinforcement.

See Commentary for Section 6.3.4.

SECTION 7

STEEL DESIGN

7.1 General

[Will develop if needed]

SECTION 8

SEISMIC DESIGN

8.1 Performance Requirements

8.1.1 General

Seismic design of the tunnel shall conform to Caltrans BDS, augmented with pertinent provisions of project specific criteria as detailed in ***Criteria.*** The Seismic Hazard Study Geotechnical Report considered two separate seismic levels, one level for the tunnel and a separate level for the design of slopes above portals. In recognition that historically the seismic performance in the vicinity of portals has been inferior to that of the tunnel, a higher seismic level was specified for slopes above portals. A maximum credible earthquake (MCE) with a return period of 1000 years was selected for rock slopes above portals while a MCE with a return period reduced to 500 years was specified for design of the tunnel and portals. Since ***Criteria*** is only for the design of structural components of the tunnel and portals, the 500 year MCE event will be used exclusively for design for all structural components of the tunnel, portal to portal.

8.1.2 Performance Requirements

The tunnel shall remain serviceable and have only experienced "Repairable Damage" after the MCE seismic event. Serviceable is defined as providing immediate access to emergency vehicles and full access to normal traffic almost immediately.

"Repairable Damage" can be defined as allowing moderate inelastic response of the lining and portals to occur. Concrete cracking, reinforcement yield, and spalling of cover concrete is expected at this level of inelastic response. The extent of damage should be sufficiently limited to permit restoration of the structure to essentially the pre-earthquake condition without replacement of any portion of the lining or portals. Damage must be repairable within 90 days and by only requiring lane closures outside peak traffic periods.

If fexural section force Demand/Capacity (D/C) ratios do not exceed a value of 1.5 when analysis and capacity assessment is performed in accordance with these criteria, it can be assumed that performance criterion required by Section 8.1.2 has been achieved.

8.2 Definitions of Ground Motions

Ground motions used in the puesdo-static and dynamic analysis of the tunnel shall be taken from the Seismic Hazard Study Geotechnical Report. The ground motion records shall consists of three sets of 3-component ground displacement time histories that are spectrum compatible to the design seismic event. These time histories were derived from actual earthquake records containing near field effects, frequently referred to as a "velocity pulse". When performing two-dimensional analyses, only those components of the ground motions that are in the plane of the analysis shall be applied. For three-dimensional analysis, all three components of ground motion shall be applied.

See Seismic Hazard Study dated August, 2001 for a further discussion on seismicity at the project site.

Faults having a major influence on the seismic design of the tunnel are the San Andreas Fault east of the tunnel and the San Gregorio Fault west of the tunnel. The San Andreas Fault is located approximately 8 kilometers from the eastern limit of the site and is capable of generating a 8.0 moment magnitude seismic event. The San Gregorio Fault is located 3 kilometers from the western limit of the tunnel and is capable of generating a 7.5 moment magnitude seismic event. Because of the closer proximity of the San Gregorio Fault, it was determined that this fault controls the MCE event.

The design ground motions will be based on a deterministic approach using a medium attenuation relationship. However, location of the tunnel places it in close proximity to both the San Gregorio and San Andreas Fault which will generate near-field effects. In recognition that directivity effects are an important near-fault consequence, a probabilistic seismic hazard analysis will also be performed in order to guide the degree of adjustments needed to apply directivity effects in the MCE design ground motions.

8.3 Analysis for Determination of Demand

8.3.1 General

Demands on the tunnel lining and portal shall be determined by analysis of local two and three-dimensional computer models. Analysis will be performed using finite element analysis software capable of evaluating the linear and non-linear behavior of the tunnel. The effects of rock/soil-interaction shall be included.

8.3.2 Combining Service and Seismic Load Demands

Effects of service loads such as geostatic and dead load shall be combined with seismic induced displacement and loading effects. Bounding designs shall be performed when it is found that a greater demand is placed on the tunnel lining and portal without the effects of a particular service load.

8.3.3 Seismic Demands – Final Lining And Cut-And-Cover Portals

Seismic demands on the final lining and cut-and-cover portal shall be determined by pseudo-static time history analysis. Boundary conditions between the initial and final lining shall allow for the formation of gaps as well as sliding when appropriate. Any reinforced concrete member found with either tension or flexural forces exceeding the modulus of rupture combined with any present axial thrust on the section shall be modeled with adjusted section properties to represent the cracked section. Reinforced concrete members with a flexural force Demand/Capacity (D/C) ratio greater than 1.5 shall be modeled with non-linear elements.

8.4 Analysis for Determination of Capacities

8.4.1 General

Capacities of structural components shall be determined by these criteria and material strain limits. Final lining and cut-and-cover portal reinforced concrete members with a flexural force DIC ratio exceeding 1.5 shall comply with the material strain limits specified in Sections 8.4.2 and 8.4.3 of the ***Criteria*** and the detailing requirements of Section 8.4.4.

8.4.2 Allowable Concrete Strain Capacities

When significant nonlinear response of reinforced concrete members is expected, concrete strain shall be limited to 213 of the ultimate strain as determined by a constitutive stress strain model for confined concrete and 213 of the spalling strain permitted for unconfined concrete when confining reinforcement is not provided.

Mander et. al. Journal of Structural Engineering, American Society of Civil Engineers, 1988, pg 1804-1849.

8.4.3 Allowable Reinforcement Strain Values

When significant nonlinear response of reinforced concrete members is expected, strains in reinforcement shall be limited to ε_{pg} as shown in the table below. The values given in the table are to be used for evaluating moment-curvature relationships for the lining and portal structures.

Reinforcement Size	ε_u	ε_{pg}
#32 (#10) Bars and Smaller	0.12	0.08
Bars Larger Than #32 (#10)	0.08	0.05

Where: ε_u = Ultimate reinforcing steel strain

ε_{pg} = Allowable reinforcing steel strain to meet ***Criteria*** performance goals

8.4.4 Plastic Hinge Length

Reinforced concrete members with a flexural force DIC ratio exceeding 1.5 shall comply with this section. Two layers of reinforcement shall be used. Sufficient cross ties shall be provided to comply with the requirements of ***Criteria*** Section 8.4.1, but not less than a #13 (#4) bar crossties spaced 150 mm (6 in.) vertically and 300 mm (12 in.) horizontally. Analytical plastic hinge length to determine the spread of plastic curvature shall be taken as shown below. Crossties shall be placed through the analytical plastic hinge length and extend the thickness of the lining above and below the plastic hinge.

Analytical Plastic Hinge Length, $L_p = H/2$

Where: H = Thickness of final lining or thickness of slab or wall of cut-and-cover portal

8.5 Seismic Design Detailing Requirements

8.5.1 Construction Joints

All longitudinal reinforcement shall be continuous through construction joints.

SECTION 9

COMMENTARY

3.2.1 Concrete

Designs should be initially based on an assumed concrete strength of $f'_c = 28.0$ MPa (4000 psi) and should only be increased to a maximum of $f'_c = 35.0$ MPa (5000 psi) if advantages such as the elimination of shear reinforcement are possible through the use of higher strength concrete.

6.3.2 Distribution of Flexural Reinforcement

When checking the flexural crack width of reinforced concrete members, a "z" factor of 22.8 Nlmm (130 kipslinch) was chosen to limit crack width to approximately 0.3 mm (0.01 in.). This criterion is consistent with standard practice for many other tunnels that utilize a double shell lining system consisting of both initial support and final lining separated by a waterproofing and drainage system. A "z" factor of 22.8 N/mm (130 kipslinch) is also used in place of the 17.2 Newton/mm (98 kipslinch) specified in BDS Section 17.6.4.6 for reinforced concrete cast-in-place boxes. The higher value was judged acceptable due to the use of a waterproofing membrane at the exterior surface of both tunnels and cut-and-cover portals.

6.3.4 Minimum Thickness of Tunnel Final Lining and Portal Slabs and Walls

The use of two layers of reinforcement at the tunnel final lining was controversial. The design team recommended the use of two layers for the following reasons:

- Key block loading occurs over much of the tunnel length and generally resulting in tensile stresses at the outside (rock) face of the final lining. Reinforcing steel was added to resist these tensile stresses. For key block loading to occur, the assumption specified in criteria section 5.2 was enforced, which assumed that the initial support deteriorated over time and that the final lining would be required to support all rock loadings, including key block loading.
- There were some areas of the tunnel that produced unsymmetrical rock loadings. This unsymmetrical loading necessitated two layers of reinforcement at these areas. Two layers of reinforcement were also required at all areas of discontinuity such as the junctions of the tunnel with cross passages.
- Final lining distortions from seismic ground displacements frequently caused tension stresses at the outside lining face under some combinations of lining self-weight and geostatic loading. The design team determined that predictable as well as very good seismic performance consisting of essentially elastic behavior could be obtained by adding reinforcement at the outside of the lining to resist these seismic displacement induced tensile stresses. Because of the close proximity to major seismic faults, the design team was also concerned that seismic ground motions would exacerbate key block loading. Since considerations listed in the first two bullet item required two layers of reinforcement over approximately 60% of the tunnel, using two layers throughout the tunnel would only result in nominal additional cost, but will also provide the contractor with a uniform tunnel section throughout the tunnel.

Technical Advisory Panel members (TAP) were generally opposed to the use of two layers of reinforcement. They took exception to the design criteria assumption that the initial support would not be used to support long-term loads. If rock dowels used as part of the initial support were assumed to be permanent, then the key block loading causing tensile stresses at the outside of the lining would never develop. This would therefore negate one of the design team's primary factors in recommending two layers of reinforcement.

The design team did not make provisions for the long-term performance of the initial support. For example, rock bolts would not be grouted to provide for long term corrosion protection. It was the design team's opinion that this would be more costly than using a more robust final lining to resist all rock loading. Proposals to use a percentage of the initial lining support to account for possible long term environmental degradation were discussed, but the design team could not reach a consensus on a rational means of estimating this percentage. The design team's final recommendation was to therefore ignore the long-tern contribution of the initial support.

TAP members were also generally opposed to proving a second layer of reinforcement for any consequences of seismic distortions. Their opinion was that if the risk of collapse was minimal and life-safety could be achieved with one layer of reinforcement, then any additional cost was not warranted.

Caltrans eventually elected to adopt the design team's recommendation and use two layers of reinforcement. However, Caltrans stipulated that the decision was made for the Devils Slide project only and that Caltrans would need to make a similar decision on future tunnels on a case-by-case basis. Caltrans stated that the deciding factor leading to their decision for Devils Slide Tunnel was the close proximity of the tunnel to major active seismic faults.

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